January 27, 1987

Department of Engineering
City Hall-Seventh Floor
455 North Main
Wichita, Ks 67202

Subject: Vacation Case V-1438
Minimum Building Pad Elevation
Miles Lakewood Living Addition
(Sandpiper Bay)

Attention: Mr. Mike Lindebak, P.E., City Engineer

Dear Mike:

Transmitted with this letter is my Report on the proposal to vacate the platted Minimum Building Pad Elevation of 1308 at the Sandpiper Bay project and substitute the elevation 1305 for future buildings. As discussed with you, my recommendation is based on updating the data used by Engineering Testing Company to calculate the seepage level when the plat was filed in 1974. The calculations and text of the ETC report is appended to my report for easy reference.

If you have any questions about the report, or if further information is desired, please advise.

Yours truly,

enc. Report (2)

M. S. Mitchell
Flood Plain Management & Land Development Specialist
1215 Forest • Wichita, KS. 67203
(316) 265-9812

REPORT ON CHANGING THE MINIMUM BUILDING PAD ELEVATION

AT

SANDPIPER BAY

TABLE OF CONTENTS

REPORT ON CHANGING THE MINIMUM BUILDING PAD ELEVATION

AT SANDPIPER BAY

In December of 1974, the Engineering Testing Company prepared a report titled "Investigation of Water Infiltration for Site North of Eighth & West of I-235 in Wichita, Kansas" which outlined the procedure followed in running soils tests and preparing a flow-net for the area now known as Sandpiper Bay. That report, based on the relationship of the channel of Big Slough-Cowskin Floodway and the Grosshans-Peterson borrow pit as it was shown on a 1970 aerial photo-enlargement, determined that the water level in the borrow pit would reach elevation 1304 under design flood conditions prescribed by the City-County Flood Control Office.

The Flood Control Office, reacting to erosion of river and channel bank lines during the spring and fall floods of 1973, suggested that the report consider the possibility that the distances shown on the 1970 photo-enlargement might be greater than they were in 1974, and greater than it might be able to guarantee to maintain. Engineering Testing then reduced the distances from the channel to the borrow pit by a uniform amount of 100 feet and recalculated the elevation which the water in the borrow pit would reach under the design flood condition. The result of the recalculation was to recommend that the Minimum Building Pad elevation for building sites bordering the borrow pit be set at 1308 based on an expected water surface elevation of 1305 for the design flood condition.

That recommendation was based on the assumption that the water level in the borrow pit would be at elevation 1302 at the beginning of the design flood, that the design flood elevation opposite the borrow pit would reach 1318.5 and that it would stay at that elevation for a period of 7 days before beginning to receed.

The City-County Flood Control Office has measured the elevation of the groundwater in an observation well immediately adjacent to the Grosshans-Peterson borrow pit since early 1973. The level in the observation well has been compared to the level in the borrow pit often enough to consider them to be the same. The Flood Control record shows that, during the period from 1973 to date, the normal water level varies approximately 0.5 feet above and below elevation 1300 and the level responding to floods in the Floodway has exceeded elevation 1301.8 only once, in October of 1973 after a month-long period when no readings were recorded. The reliability of the readings for September-October 1973 is doubtful based on the history of the succeeding 13 years.

At the time of the Report by Engineering Testing Company, the only tool which the Flood Control Office had to estimate the maximum flood elevation was to assume that the levee crest was a set freeboard distance above the design flood elevation. Using that method, the elevation 1318.5 was used in the ETC report. Since that time, the City has had the benefit of a study by the Federal Insurance Administration which determined that the level of the 100 year frequency flood for the location under study is 1316. Duration of the crest of that design flood used in the ETC report was assumed to be 7 days. No justification for that assumption has been discovered and it is interesting to note that if the 100 year frequency discharge were to crest at elevation 1316 and remain steady for 7 days the total volume of floodwater passing thru the Floodway would be approximately 500,000 acre feet while the total volume of the 1979 flood, the record for the Floodway, was only in the 70,000 acre foot range. It is apparent that the 7 day duration for the design flood crest is too long.

In the period between 1975 and now, the City-County Flood Control Office has pursued an active program of using building and paving demolition spoil and rubble to stabilize the channel banks of various waterways and channels under their maintenance jurisdiction. The left (east and south) bank of the Floodway between Central and Zoo Boulevard has been stabilized under that program and the distance between the toe of the Floodway channel and the centerline of levee now ranges from 300 to 350 feet. In addition to stabilizing the distance, the substitution of a 15 to 50 foot layer of nearly impervious material has reduced the potential for seepage from the channel into the borrow pit now called Sandpiper Bay.

When the ETC report was filed, the uncertainty of being able to maintain a levee foreshore distance of 300 feet or more, and the lack of record on the amount of seepage which might be expected in the borrow pit during floods, dictated that a generous freeboard, or safety factor, be included in the calculations to set the Minimum Building Pad elevation for future buildings. With the entry of the City into the Regular Flood Insurance Program via an Ordinance which specifies a freeboard of only one foot above the calculated level of the 100 year frequency flood, it seems appropriate to apply the same freeboard to the elevation of the calculated level of seepage into Sand Piper Bay from a flood which reaches the 100 year flood level in the adjacent Floodway.

With the above conclusions in mind, the method of calculating the seepage elevation employed by ETC in 1974 was applied to the relationship of the Floodway channel and Sand Piper Bay as shown on the 1983 edition of photo-enlargement 5048 and to the 100 year flood level in the Floodway. For this exercise two cases were analyzed, one with the seepage path distance measured from the top of channel bank as shown on the photo-enlargement, and the other case measuring the same distance from the edge of vegetation at the toe of the channel bank slope. Comparison of the results of the two flow-net models tests the sensitivity of the seepage path determination.

Data from Tables I and II of the ETC report were used without any change. Minor clerical errors were found in Table III B and corrections used to determine the total weighted average factor "k" at the bottom of Table III D. Using the photo-enlargement, a new Table IV was prepared for the two seepage path cases and used to obtain two values of Nf/Nd at the bottom of Table V. Because the area around the water surface of Sandpiper Bay has been changed from the outline of the old borrow pit which ETC measured in 1974, a new surface area was measured and used in calculations for Table VI. Again, two seepage path models were used to prepare Table VI. Comparison of the results of using the two models shows that in neither case would the seepage level in Sandpiper Bay reach elevation 1304 in the 7 day flood duration.

CONCLUSIONS AND RECOMMENDATION

1. The results of the ETC method of calculating the seepage elevation in Sandpiper Bay are not radically sensitive to differences in the measured seepage path distances between the Floodway channel and the Bay water surface. Using aerial photo-enlargements to model the seepage paths is well within the range of other factors used in the method.

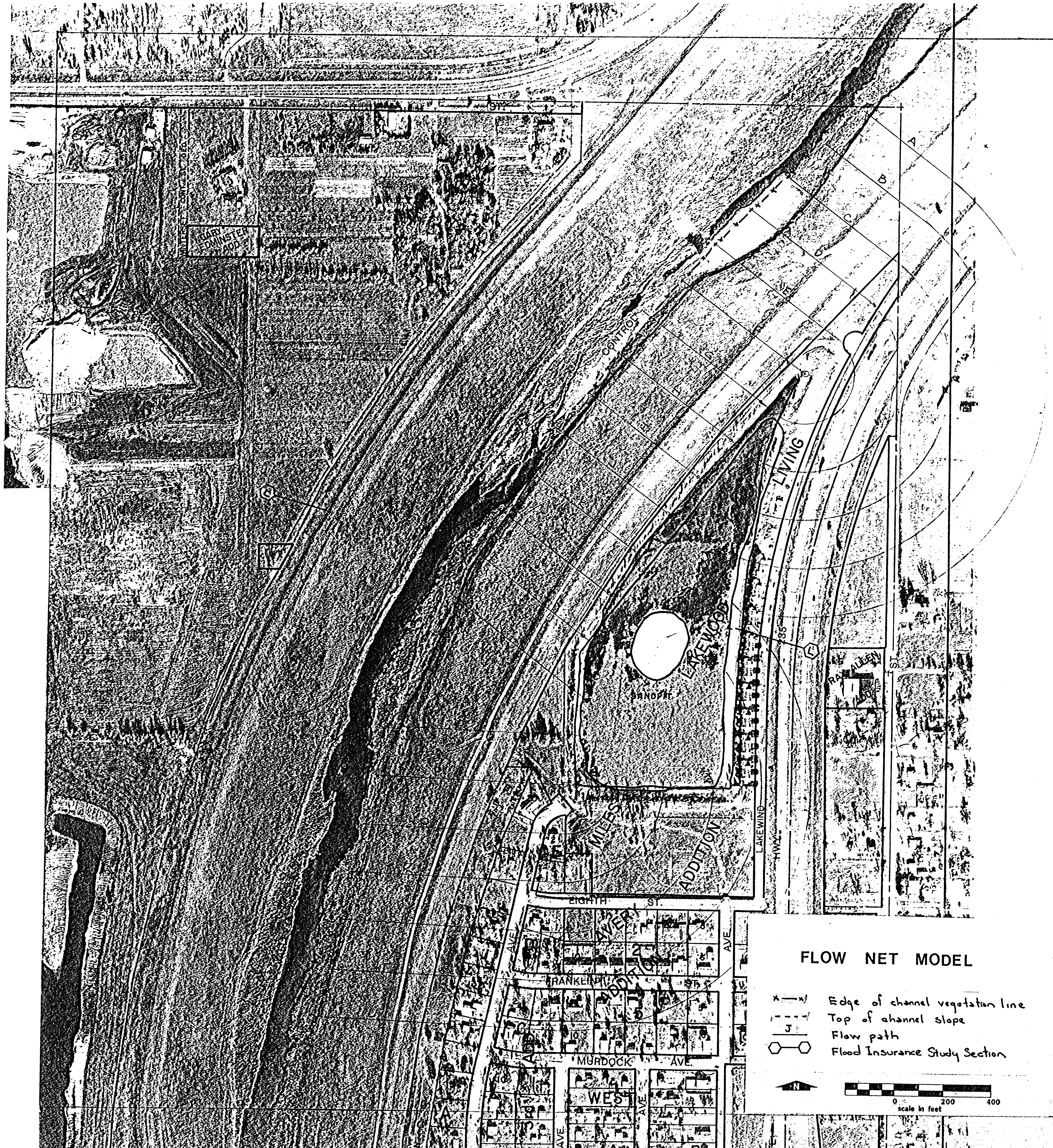
2. Using elevation 1300 as a starting water level in the Bay is consistent with the record of the Flood Control observation well for the period 1973 thru 1986.

3. Using the 100 year frequency flood level in the Floodway is consistent with the balance of the City drainage and floodplain management program.

4. The 7 day duration is probably grossly overstated, but is used in this analysis to be consistent with the ETC report.

5. One foot of freeboard over the calculated seepage level in the Bay is consistent with the City Floodplain Management Ordinance.

6. Based on the models and calculations contained in this report, vacation of the platted Minimum Building Pad Elevation of 1308 and substitution of an elevation of 1305 is recommended.


FLOW NET MODEL
Edge of channel vegetation line
Top of channel slope
Flow path
Flood Insurance Study Section
N
0
200
400
scale in feet
EIGHTH ST.
FRANKLIN
MURDOCK AVE.
LAKEWOOD
LIVING
MILES
ADDITION
LAKEWIND
HWY.
WEST
CONTROL
AVE.

NEW TABLE IV A

LENGTH OF FLOW LINES
(Measured from top of channel)

FLOW LINE	STRAIGHT SECTION (feet)	CIRCULAR RADIUS (feet)	SECTION CIRCUMFERENCE (feet)	DEGREES	CIRCULAR LENGTH (feet)	TOTAL LENGTH (feet)
A	450	1000	6283	159	2775	3225
B	440	800	5027	162	2262	2702
C	430	600	3770	161	1686	2116
D	440	400	2513	166	1159	1599
E	500	200	1257	170	593	1093
F	580					580
G	560					560
H	560					560
I	550					550
J	540					540
K	540					540
L	540					540
M	590					590
N	650					650
O	730					730
P	760					760
Q	790	200	1257	87	304	1094
R	830	400	2513	90	628	1458
S	870	600	3770	93	974	1844
T	910	800	5027	129	1801	2711
U	960	1000	6283	141	2460	3420

NEW TABLE IV B

LENGTH OF FLOW LINES
(measured from toe of channel)

FLOW LINE	STRAIGHT SECTION (feet)	CIRCULAR RADIUS (feet)	SECTION CIRCUMFERENCE (feet)	DEGREES	CIRCULAR LENGTH (feet)	TOTAL LENGTH (feet)
A	540	1000	6283	159	2775	3315
B	550	800	5206	162	2262	2812
C	560	600	3770	161	1686	2246
D	590	400	2513	166	1159	1749
E	620	200	1257	170	593	1213
F	630					630
G	610					610
H	610					610
I	580					580
J	610					610
K	560					560
L	600					600
M	690					690
N	730					730
O	820					820
P	980					980
Q	850	200	1257	87	304	1154
R	940	400	2513	90	628	1568
S	980	600	3770	93	974	1954
T	1000	800	5027	129	1801	2801
U	1070	1000	6283	141	2460	3530

NEW TABLE V A
(with flow lines measured from top of channel)

CALCULATED VALUES OF Nd AND Nf/Nd

FLOW LINE	TOTAL LENGTH (feet)	Nd	Nf/Nd
A	3225	1612	.0105
B	2702	1351	.0126
C	2116	1058	.0161
D	1599	800	.0213
E	1093	546	.0311
F	580	290	.0586
G	560	280	.0607
H	560	280	.0607
I	550	275	.0618
J	540	270	.0630
K	540	270	.0630
L	540	270	.0630
M	590	295	.0576
N	650	325	.0523
O	730	365	.0466
P	760	380	.0447
Q	1094	547	.0311
R	1458	729	.0233
S	1844	922	.0184
T	2711	1356	.0125
U	3420	1710	.0099

Total Nf/Nd = .8188

NEW TABLE V B

(Using flow lengths measured from toe of channel)

CALCULATED VALUES OF Nd AND Nf/Nd

FLOW LINE	TOTAL LENGTH (feet)	Nd	Nf/Nd
A	3315	1658	.0103
B	2812	1406	.0121
C	2246	1123	.0151
D	1749	874	.0194
E	1213	606	.0280
F	630	315	.0540
G	610	305	.0557
H	610	305	.0557
I	580	290	.0586
J	610	305	.0557
K	560	280	.0607
L	600	300	.0567
M	690	345	.0493
N	730	365	.0466
O	820	410	.0415
P	980	490	.0347
Q	1154	577	.0295
R	1568	784	.0217
S	1954	977	.0174
T	2801	1400	.0121
U	3530	1765	.0096

Total Nf/Nd = .7444

NEW TABLE VI

QUANTITY AND TIME CALCULATIONS FOR
WATER INFILTRATION INTO LAKE WITH DESIGN WATER SURFACE = 1316

FINAL ELEVATION IN LAKE (feet)	AVERAGE DIFFERENTIAL HEAD (h) (feet)	VOLUME OF WATER Qt=17.24h (cfm)	TIME FOR 1 FOOT RISE (minutes)	TIME FOR 1 FOOT RISE (days)	CUMULATIVE TIME FOR RISE (days)
(With flow lines measured from top of channel)					
1301.0	15.5	267	2294	1.59	1.59
1302.0	14.5	250	2450	1.70	3.29
1303.0	13.5	233	2628	1.82	5.11
1303.96					
					7.00
1304.0	12.5	216	2835	1.97	7.08
1305.0	11.5	198	3093	2.15	9.23
1306.0	10.5	181	3383	2.35	11.58
(With flow lines measured from toe of channel)					
1301.0	15.5	243	2520	1.75	1.75
1302.0	14.5	227	2698	1.87	3.62
1303.0	13.5	212	2889	2.01	5.63
1303.6					
					7.00
1304.0	12.5	196	3124	2.17	7.80
1305.0	11.5	180	3328	2.31	10.11
1306.0	10.5	165	3723	2.59	12.70

ENGINEERING TESTING COMPANY
WICHITA KANSAS

GENE MILES, REALTOR
WICHITA KANSAS

SCALE: 1" = 200'	APPROVED BY:	DRAWN BY:
DATE: 12-5-74		REVISED

PLOT PLAN
N.W. CORNER BIGHTIN & I235

WICHITA KANSAS

DRAWING NUMBER: FIG #1

ENGINEERING TESTING COMPANY
535 North Washington / (316) 265-8553
P. O. BOX 2391 / WICHITA, KANSAS 67201

ORVAL W. DANIELS
REGISTERED PROFESSIONAL ENGINEER

R. A. ROTHHAMMER
GEOLOGIST

December 18, 1974

Gene Miles, Realtor
10711 West Kellogg
Wichita, Kansas

Dear Mr. Miles:

At the property north of Eighth Street and between I-235 Bypass and the Wichita Valley Center Flood Control Ditch, I have assumed that the pilot channel now located near the center of the ditch has moved 100 feet east of its present location. The calculations for inflow into the lake have been modified to reflect this change. If all of the flow line lengths are shortened 100 feet, the value for Total N_f/N_d = 0.8886 and Q_t = 16.30 h. Under design flood conditions, the time required for the water to rise from elevation 1300.0 feet to 1304.0 feet, would equal 7.619 days.

If it is assumed that the initial level of the water in the lake is at elevation 1302.0 feet, the water would rise to elevation 1305.0 feet in 6.273 days. The water surface elevation would be at elevation 1305.3 feet at the end of 7 days, the duration of a design flood.

If the pilot channel of the ditch moves 100 feet toward the lake, the water level would rise from elevation 1302.0 feet to 1305.0 feet in 6.273 days instead of in 7.273 days as calculated previously.

The shifting of the pilot channel toward the lake will affect the time required for the water to rise in the alke, but this change in time is not great. The water levels achieved in the lake after any flood would be slightly higher if the pilot channel moves, but the differences in elevation would be minor.

The calculations for lengths of flow lines, N_d and N_f/N_d are shown by table VII and the times required for water infiltration into the lake are shown by table VIII and are attached hereto:

Respectfully submitted,

ENGINEERING TESTING COMPANY

Orval W. Daniels, P.E.

OWD:du

cc: 4 to Gene Miles, Realtor
1 to file

Calculated Values of N_f/N_d

Assume Pilot Channel in Drainage Ditch moves 100 feet East of its present location

Flow Line	Length	N_d	N_f/N_d
A	3,190	1,595	.0107
B	2,682	1,341	.0127
C	2,250	1,125	.0151
D	1,687	844	.0201
E	1,123	562	.0302
F	550	275	.0618
G	510	255	.0667
H	510	255	.0667
I	470	235	.0723
J	520	260	.0654
K	530	265	.0641
L	500	250	.0680
M	480	240	.0708
N	560	280	.0607
O	630	315	.0540
P	690	345	.0493
Q	1,013	507	.0335
R	1,398	699	.0243
S	1,774	887	.0192
T	2,630	1,315	.0129
U	3,350	1,675	.0101

Total N_f/N_d = .8886

Table VII

Quantity and Time Calculations

Water Infiltration into Lake (Area = 734,600 sq.ft.)
Design High Water Elevation 1318.5 feet
Pilot Channel in Ditch 100 feet East of Present Location

Final Water Elevation in Lake (Feet)	Average Head Differential (h) (Feet)	Volume of Water (Cu.Ft./Min.) $Q_t = 16.30\ h$	Time for 1 Foot rise (Minutes)	Time for 1 Foot Rise (Days)	Cumulative Time for Water rise (days)
1301.0	18.0	293	2507	1.741	1.741
1302.0	17.0	277	2652	1.842	3.583
1303.0	16.0	261	2814	1.954	5.537
1304.0	15.0	245	2998	2.082	7.619
1305.0	14.0	228	3221	2.237	9.856
1306.0	13.0	212	3465	2.406	12.262
1307.0	12.0	196	3748	2.603	
1308.0	11.0	179	4103	2.849	
1309.0	10.0	163	4507	3.130	
1310.0	9.0	147	4997	3.470	
1311.0	8.0	130	5650	3.924	

Table VIII

etc

Received DEC 11 1974

Investigation of Water Infiltration

for

Site N. of Eighth & W. of I-235

Wichita, Kansas

for

Gene Miles

Wichita, Kansas

ENGINEERING TESTING COMPANY
535 North Washington / (316) 265-8553
P. O. BOX 2391 / WICHITA, KANSAS 67201

ORVAL W. DANIELS
REGISTERED PROFESSIONAL ENGINEER

R. A. ROTHHAMMER
GEOLOGIST

December 9, 1974

Mr. Gene Miles
10711 West Kellogg
Wichita, Kansas

Attn: Mr. Gene Miles

Dear Mr. Miles:

We have drilled four test holes on your property located north of Eighth Street and between the I-235 Bypass and the Wichita-Valley Center Flood Control Ditch.

The logs of the test holes, results of the tests made and the calculation of water inflow into the lake during periods of high water are attached hereto:

Respectfully submitted,

ENGINEERING TESTING COMPANY

Orval W. Daniels, P.E.

OWD:du

cc: 4 to Gene Miles
1 to file

Investigation of Water Infiltration

at

Lake

North of Eighth & West of I-235

Wichita, Kansas

for

Gene Miles

Wichita, Kansas

by

Orval W. Daniels, P.E.

Four test holes were drilled near the lake, which is located north of Eighth Street and between I-235 Bypass and the Wichita-Valley Center Flood Control Ditch, Wichita, Kansas. The locations of the test holes are noted on the logs and are shown by figure #1. The benchmark used to determine the elevations of the tops of the test holes was a railroad spike driven into the power pole located at the northeast corner of Arapaho and Franklin Streets. The elevation of the benchmark equals 1311.84 feet.

The test holes were drilled using water and bentonite to remove the cuttings from the holes. During the course of the drilling, standard penetration tests were conducted by driving a 2 inch outside diameter split spoon sampler, with a 140 pound hammer dropped 30 inches. The number of blows required to drive the sampler each 0.5 foot increment is recorded in the logs. The logs of the test holes and the number of blows required to drive the sampler the final foot, are shown graphically by figure #2. The samples secured in the split spoon sampler, were used for the grain size tests of the soil. All of the test holes, were completed with penetration into the Wellington Shale. The elevations of the tops of the test holes, the elevation of the top of the sand, the elevation of the top of the shale and the depth of sand at each location are shown by table I.

Test Hole Number	Elevation Top of Ground (Feet)	Elevation Top of Sand (Feet)	Elevation Top of Shale (Feet)	Depth of Sand (Feet)
1	1311.8	1308.8	1274.8	34.0
2	1311.2	1308.4	1274.2	34.2
3	1313.1	1306.1	1274.1	32.0
4	1313.0	1310.0	1275.4	34.6

Table I

This site is underlain by sandy clay to a depth of approximately 3 feet, sand to a depth of approximately 37 feet and by clayey shale to the bottom of the test holes. The sand is reasonably uniform in size with coarser material found in test hole #1, at elevation 1292 feet and near the bottom of the test hole, at elevation 1277 feet.

Wash loss and sieve analysis tests were conducted on 15 samples of sand secured from the test holes. Grain size analysis tests were conducted on two samples of the clayey shale, from the bottom of the test holes by hydrometer. The sandy clay soil to a depth of 3 feet and the clayey shale were considered so impervious compared to the sand that no appreciable amount of water would flow through these materials. The wash loss and sieve analysis tests are reported by laboratory numbers 113695 through 113709 and the grain size analysis tests are reported by laboratory numbers 114169 and 114170. The rate of flow of water through the soil is dependent upon the grain size of the soil where 10% of the soil is finer. The size of the grains where 10% passes was determined from the Size Distribution Charts with this size for each sample expressed in millimeters. Darcy's coefficient of permeability may be calculated by the expression, $k = 150\ (D_{10})^2$: where d_{10} is expressed in centimeters and "k" in centimeters per second. The values of "k" are converted to flow in feet per minute by multiplying the centimeter per second value by 2.

The values for D_{10} for the sand, were determined from the Size Distribution Charts of the wash loss and sieve analysis tests and are tabulated by table II. The thickness of the sand layer represented by each gradation test was determined and multiplied by the appropriate "k" value. The sum of these products was then divided by the total thickness of the sand stratum and the weighted average value of "k" for each test hole was determined. These calculations are shown by table III A, III B, III C and III D. The average value of "k" was determined by averaging the weighted averages of k from the four test holes and equals .0917 feet per minute for the sand stratum.

The patch of least resistance to the flow of water from the Wichita-Valley Center Flood Control Ditch to the lake will be through the sand stratum between ungrassed center channel of the ditch and the sides of the lake. Because of water flow from the ditch to the lake, the silting on the banks of the lake will be considered ineffective in retarding water flow.

During periods of high water in the drainage ditch water will flow through the sand into the lake under study. The design high water elevation at Tenth Street and the drainage ditch equals 1318.5 feet, as furnished to us by the Wichita-Sedgwick County Flood Control Office. The design high water elevation is assumed to continue for 7 days.

A number of conditions have been assumed for this study and they include the following:

Values for D_{10} and k for Sand

Hole No.	Elevation (Feet)	d_{10} (mm)	d_{10} (cm)	d_{10}^2 (cm^2)	$150(d_{10})^2$ cm/sec	k (feet/min.)
1	1306.8	.180	.0180	.00032	.0480	.0960
1	1301.8	.070	.0070	.00005	.0075	.0150
1	1291.8	.355	.0355	.00126	.1890	.3780
1	1286.8	.175	.0175	.00031	.0465	.0930
1	1281.8	.240	.0240	.00058	.0870	.1740
1	1276.8	.600	.0600	.00360	.5400	1.080
2	1301.2	.053	.0053	.00003	.0045	.0090
2	1291.2	.165	.0165	.00027	.0405	.0810
2	1281.2	.180	.0180	.00032	.0480	.0960
3	1303.1	.135	.0135	.00018	.0270	.0540
3	1293.1	.106	.0106	.00011	.0165	.0330
3	1283.1	.090	.0090	.00008	.0120	.0240
4	1303.0	.098	.0098	.00010	.0150	.0300
4	1293.0	.149	.0149	.00022	.0330	.0660
4	1283.0	.165	.0165	.00027	.0405	.0810

Table II

Weighted Average Value of k
Test Hole #1

k	Layer Thickness (Feet)	Product
.0960	5.0	.4800
.0150	7.5	.1125
.3780	7.5	2.8350
.0930	5.0	.4650
.1740	5.0	.8700
1.0800	4.0	4.320
Total	34	9.0025 ft./min.

Weighted Average k = 9.0025/34 = .2648

Table III A

Test Hole #2

k	Layer Thickness (Feet)	Product
.0900	12.0	1.080
.0810	10.0	.810
.0960	12.0	1.152
Total	34	3.042 ft./min.

Weighted Average k = 3.042/34 = .0895

Table III B

Weighted Average Value of k
Test Hole #3

k	Layer Thickness (Feet)	Product
.0540	8.0	.432
.0330	10.0	.330
.0240	14.0	.336
Total	32.0	1.098

Weighted Average k = 1.098/32 = .0343 ft./min.

Table III C

Test Hole #4

k	Layer Thickness (Feet)	Product
.0300	12.0	.360
.0660	10.0	.660
.0810	12.5	1.013
Total	34.5	2.033

Weighted Average k = 2.033/34.5 = .0589 ft./min.

Table III D

Total Weighted Average "k" = .0917 ft./min.

1. No water will enter the project over the tops of the dikes from the drainage ditch.
2. The surface drainage pattern of the surrounding area will not be fundamentally changed to discharge additional water into the lake.
3. The general layout of the lake will be fundamentally as shown after completion. If the lake area is increased, the water rise will be slower. If the size of lake is decreased significantly, the rate of water rise will probably be more rapid.
4. Figure #1 was drawn using as a guide Aerial Photograph #5048 prepared by Kucera and associates for Sedgwick County Area Metropolitan Planning Commission and K.G.&E., the Electric Company in February 1968. This lake was excavated as a borrow pit for I-235 to a depth approximately 8 feet below the present water surface.
5. A survey to determine the elevations of the top of the test holes was made on November 18, 1974. At that time, the elevation of the water surface at the lake was 1300.04 feet and the elevation of the water surface in the drainage ditch was 1299.71 feet.
6. The silt on the sand in the drainage ditch in the usual flow channel is assumed to be taken into suspension during periods of high water. At that time the water will impinge on the sand.
7. The sandy silt inside the floodway, which is supporting vegetation, will be assumed to remain in the same condition during floods. The silt is sufficiently impervious that the rate of water flow will be slower than through the sand.
8. The straight lengths of the various flow lines were scaled from the plan and the curved lengths were calculated from the radii.

The flow lines were drawn at intervals of 200 feet.

9. The shale below elevation 1275 feet is assumed to be impervious.

The flow lines for water entering the lake from the drainage ditch designated A through U are shown by figure #1. The lengths of the flow lines are shown by table IV. The value of N_d equals the length of the flow lines divided by 2 and the values for N_f equals the depth from the top of the sand to the top of the shale and equals 34/2 = 17. The values for N_f/N_d are calculated for each flow line by table V. The summation of the N_f/N_d values equals 0.7658.

The seepage flow through the soil may be calculated along each flow line by the expression $Q = h\ k\ N_f/N_d\ 200$: where Q equals the flow in cubic feet per minute, h equals the difference in head between the two bodies of water; k equals the average of Darcy's Coefficient in feet per minute, N_f equals the number of flow channels in a vertical direction; N_d equals the number of potential energy drops along the flow channels and 200 feet is the width of soil section represented by each flow line. N_f and N_d each represent a distance of 2 feet as the flow lines and drop of potential lines form squares.

Since each flow line represents a horizontal distance of 200 feet, the value of the total amount of water inflow, $Q_t = 200\ h\ k$ multiplied by the summation of N_f/N_d. Q_t will vary only with "h" and $Q_t = 14.045\ h$. The surface area water in the lake equals 734,600 square feet. The surface area of the water will become somewhat greater as the water rises because the banks of the lake slope away from the water, but no correction was made for increase of water surface. The initial water elevation in the lake is assumed to equal 1300.0 feet and the average "h" for a rise to

Length of Flow Lines

Flow Line	Straight Section (Feet)	Circular Radius (Feet)	Section Circumference (Feet)	Degrees	Circular Length (Feet)	Total Length (Feet)
A	480	1,000	6,283	161	2,810	3,290
B	520	800	5,026	162	2,262	2,782
C	570	600	3,770	165	1,780	2,350
D	600	400	2,513	170	1,187	1,787
E	630	200	1,256	170	593	1,223
F	650					650
G	610					610
H	610					610
I	570					570
J	620					620
K	630					630
L	600					600
M	580					580
N	660					660
O	730					730
P	790					790
Q	830	200	1,256	81	283	1,113
R	870	400	2,513	90	628	1,498
S	900	600	3,770	93	974	1,874
T	930	800	5,026	129	1,800	2,730
U	990	1,000	6,283	141	2,460	3,450

Table IV

Calculated Values of N_d and N_f/N_d

Flow Line	Total Length (Feet)	N_d	N_f/N_d
A	3,290	1,645	.0103
B	2,782	1,391	.0122
C	2,350	1,175	.0145
D	1,787	893	.0190
E	1,223	612	.0277
F	650	325	.0523
G	610	305	.0557
H	610	305	.0557
I	570	285	.0596
J	620	310	.0548
K	630	315	.0540
L	600	300	.0567
M	580	290	.0586
N	660	330	.0515
O	730	365	.0466
P	790	395	.0430
Q	1,113	557	.0305
R	1,498	749	.0227
S	1,874	937	.0181
T	2,730	1,365	.0124
U	3,450	1,725	.0099

Total N_f/N_d = .7658

Table V

Quantity and Time Calculations for

Water Infiltration into Lake
Design High Water Elevation = 1318.5 Feet

Final Water Elevation in Lake (Feet)	Average Head Differential (h) (Feet)	Volume of Water (Cu.Ft./min.) $Q_t = 14.045h$	Time for 1 foot rise (minutes)	Time for 1 foot rise (days)	Cumulative Time for water rise (days)
1301.0	18.0	253	2,903	2.016	2.016
1302.0	17.0	239	3,073	2.134	4.150
1303.0	16.0	225	3,265	2.267	6.417
1304.0	15.0	211	3,481	2.417	8.834
1305.0	14.0	197	3,728	2.589	11.423
1306.0	13.0	183	4,014	2.787	14.210
1307.0	12.0	169	4,347	3.019	
1308.0	10.0	140	5,247	3.644	
1309.0	9.0	126	5,830	4.049	
1310.0	8.0	112	6,559	4.554	
1311.0	7.0	98	7,496	5.206	

Top of Ground

Table VI

1301.0 feet equals 1318.5 - 1301.0 + .5 = 18.0 feet. The time required for the water to rise each one foot increment has been calculated by table VI. The last column in this table shows the time required for the water to attain the elevation shown.

If the initial water elevation in the lake is 1300.0 feet, the water elevation in the lake would be 1304.0 feet after 8.834 days and the maximum waterelevation would be between 1303 and 1304 feet after a maximum high water of 7 days duration. If the initial water elevation was 1302 feet in the lake, the final water elevation would be 1305 feet after maximum high water during 7.273 days. It would appear safe to assume that the initial water elevation in the lake and in the drainage ditch was at 1302 feet prior to maximum design water elevation. To achieve elevation 1302 feet in the lake would require that a considerable amount of water would flow in the ditch for a long period of time to allow the two water elevatinns to stabilize. The maximum design water flow would then need to occur and remain at design high water level for 7 days.

At a calculated water elevation in the lake of 1305 feet, the lowest floor elevations in buildings adjacent to the lake should not be below 1308 feet to allow for 3 feet of freeboard above the water.

If possible, the surface drainage for the project should be channeled south away from the lake. Sewer lines, either sanitary or storm sewers, placed below elevation 1305 feet, should be constructed with water tight joints to prevent infiltration into the sewers.

Respectfully submitted,

ENGINEERING TESTING COMPANY

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OWD:du

cc: 4 to Gene Miles
1 to file